27 Apr 1981, 10:30 am - 1:00 pm

Session 1: Discussions and Replies

Multiple Authors

Discussion by Adel Saada,
Dept. of Civil Engineering,
Case Western REserve University,
on "Strength of Soils".

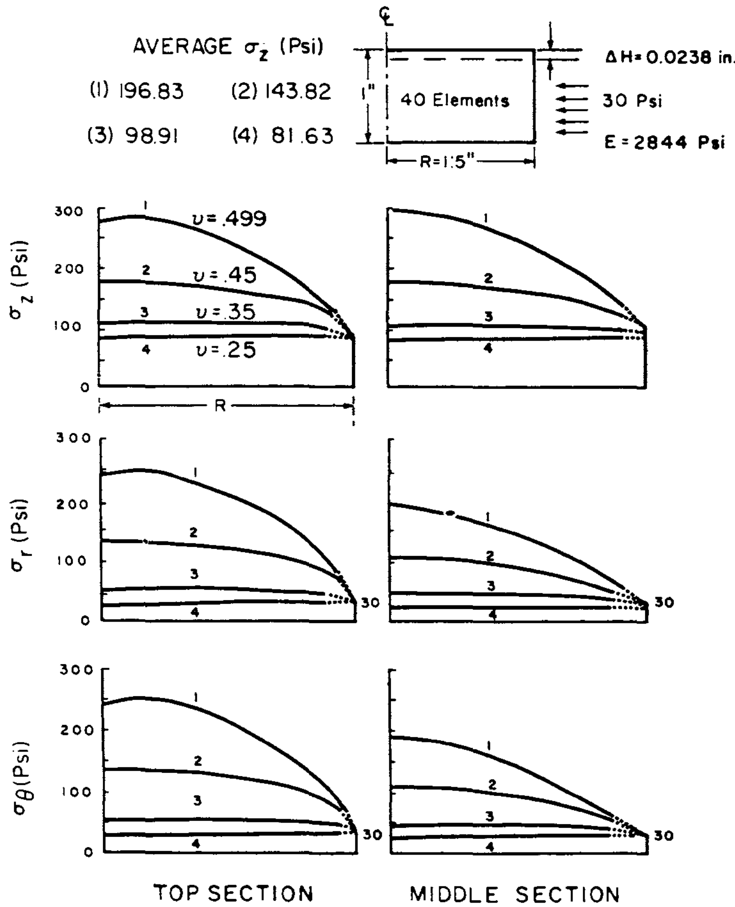

Fig. 2. Distribution of the Normal Stresses in A Simple Shear Device Under Axial Loading

Brief Discussion on The Stress Distribution In The Simple Shear Device

A comparative study of the results obtained with various testing apparatuses has been in progress at Case Western Reserve for nearly a year. Emphasis has been placed on the N.G.I. simple shear device (SS) the standard triaxial apparatus (ST) and the thin long hollow cylinder (HC). The distribution of shearing stresses in the simple shear device has been studied by the finite element method as well as photoelastically and found to be quite non-uniform.

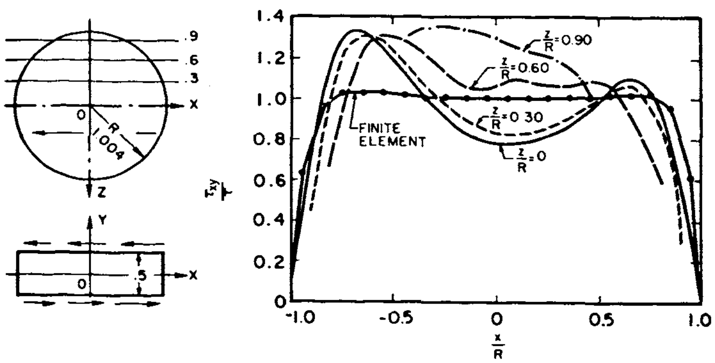

Fig. 1. Distribution of the Shearing Stresses on the Central Plane of the Specimen in a Simple Shear Test as Determined by Three Dimensional Photoelastic Techniques

Fig. 1 from Saada and Townsend, 1980 shows the distribution of the shearing stresses on the central plane of the specimen based on an elastic analysis. Just as serious is the lack of uniformity in the axial, radial and circumferential normal stresses when the specimen is subjected to axial loads alone in a wire reinforced membrane, or to axial loads in a pressurized cell. The top and bottom platens of the simple shear device are rigid and often have ribs (or similar arrangements) to avoid slipping. Under such conditions there is often very little relation between the normal stresses acting on the lateral surface of the sample and σ_r and σ_θ within the specimen. The physical dimensions as well as the boundary conditions prevent any sort of uniformity to prevail. Fig. 2 illustrates the results of a finite element analysis conducted on a specimen of standard size subjected to an axial displacement of 0.0238 in. and a lateral cell pressure of 30 psi. Young's modulus is chosen to be 2844 psi. The average vertical stress varies with the value of Poisson's ratio ν. However, more important is the fact that σ_r and σ_θ are indeed quite different from the 30 psi applied by the cell pressure. Actually, depending on the value of Poisson's ratio σ_r and σ_θ can be in the larger part of the specimen closer to σ_z than they are to the laterally applied stress. For example, Fig. 2 shows that, close to the center in the middle section, σ_z = 293.28, and $\sigma_\theta = \sigma_r$ = 187.84 psi for ν = 0.499; those values drop to 103.75 and 49.49 psi respectively for ν = 0.35. Therefore, only a very thin layer close to the lateral faces "sees" the applied 30 psi.

The results presented in Fig. 2 lead one to examine more thoroughly the process of normalization. For example in liquefaction tests normalization was traditionally made with respect to the vertical effective stress $\bar{\sigma}_{vc}$. When those results failed to coincide with those obtained from triaxial test, normalization was made with respect to the octahedral normal stress; that resulted in essentially doubling the value τ/σ_{vc} and bringing in line the results of simple shear test and triaxial test. In fact, Fig. 2 shows that there is practically an infinite number of ways that researchers can use to bring the results of the tests mentioned above to coincide: Just chose the appropriate ν. Consider for example ν = 0.499, and an average τ of 20 psi. Then

$$\frac{\tau}{\bar{\sigma}_{vc}} = \frac{20}{196.83} = 0.106 \text{ and } \frac{\tau}{\bar{\sigma}_{oct}} = \frac{20}{(196.83 + 30 + 30)/3} = 0.23;$$

$$\text{for } \nu = 0.45, \ \frac{\tau}{\bar{\sigma}_{vc}} = 0.14, \ \frac{\tau}{\bar{\sigma}_{oct}} = 0.29; \text{ and so on.}$$

In addition those numbers are still quite misleading since they use an average vertical stress and a lateral stress which may have very little in common with the values of σ_r and σ_θ acting inside the specimen.

A second point of interest in the use of the simple shear device is the assumption that the change in the axial stress in a constant volume test is equal to the pore water pressure that would have developed in an undrained test. Taylor conducted constant volume tests by changing the spherical pressure in a triaxial cell and considered that this change was equal to the pore water pressure that would have developed if the test was of an undrained nature. In a triaxial test such an assumption is acceptable since a spherical system of stress is used to counterbalance the hydrostatic pressure in the pores; on the other hand it seems difficult to accept the assumption that the change in the axial stress in the simple shear device is equal to the pore water pressure when the lateral stresses do not vary by the same amount. To investigate this point, as well as to evaluate the differences among the results obtained in the triaxial test, the hollow cylinder test and the simple shear test (Fig. 3) an experimental program was conducted on a Kaolinite clay of known mechanical properties. This clay, Edgar plastic Kaolin has been widely used in many of the published research (2,3,4). Hollow cylinder tests were conducted in a special pneumatic analog computer (SPAC) described in references 2 to 4. A special simple shear machine, very rigid and which allows accurate measurements of forces and displacements was designed and built in our laboratories for the purpose of this investigation.

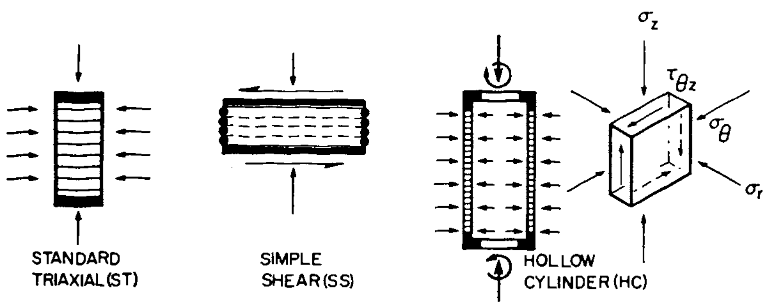

Fig. 3. Forces and Stresses Acting on the Three Configurations Examined in the Experimental Program

Experimental Program

a) Static and dynamic tests were conducted in the triaxial cell on K_o consolidated clay. The excess axial load needed to provide this type of consolidation was released approximately 12 hours before the start of the test and the sample allowed to rebound under the cell pressure and the applied back pressure of 18 psi. The shear modulus obtained from the standard triaxial test is based on the assumption that Poisson's ratio is equal to 0.5, the maximum shearing stress given by $(\sigma_1 - \sigma_3)/2$ and the maximum shearing strain given by $1.5\varepsilon_a$, where ε_a is the axial strain

b) Two series of static and dynamic tests were conducted in the hollow cylinder device on K_o consolidated clay. In one series the axial load in excess of the cell pressure was kept constant (CF) while torsional stresses were applied and pore pressures and shearing strains measured. In another series the length of the sample was fixed (CL) at the beginning of the tests and the drop in the vertical effective stress measured as the torsional stresses were applied. Thus, a comparison between so called undrained and constant volume tests could be made. Normalization was made with respect to the vertical effective stress.

c) The same procedure used in (b) was repeated in the NGI simple shear device.

This experimental program allows one to compare the shearing stress-strain characteristics obtained in the hollow cylinder and the simple shear device both under constant vertical force (i.e., undrained) and at constant height (i.e., constant volume). It also allows one to check the validity of the assumption that the change in the vertical effective stress in a constant volume test is equal to the pore water pressure generated in an undrained test.

Three water contents were examined; however, the results obtained for one water content are the only ones reported here.

Results and Discussion

Figs. 4a and 4b show the stress-strain curves obtained from simple shear test and hollow cylinder test under constant force (undrained) and constant length (constant volume) conditions. The differences are seen to be quite substantial.

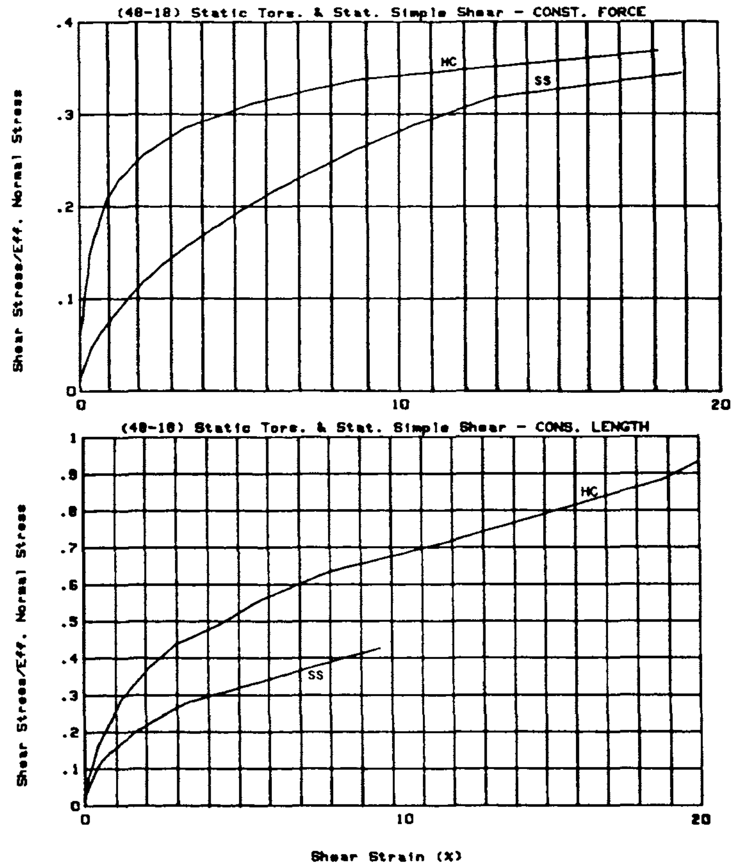

Fig. 4. Stress-Strain Curves for Constant Force and Constant Length Conditions

Fig. 5a and 5b show that whether it is the simple shear test or the hollow cylinder test the pore pressure in an undrained test is very different from the change in the axial effective stress in a constant volume test. It is obvious that the compliance of the system plays an important part as pointed out by L. Finn in a paper in this conference. There is no doubt that the device used at the University of British Columbia offers more rigidity than the reinforced membrane used in this investigation. However, we have chosen to conduct our experiment on the most common simple shear device. The results above still hold when the simple shear device is placed inside a cell and pressurized to avoid the use of the metal reinforcement of the membrane.

Fig. 6 shows the shear modulus (secant modulus) obtained from the various devices, the highest being the one obtained from the standard triaxial test. Notice that for the same apparatus the modulus is different depending on whether the test is undrained or constant volume. The differences become quite high for small strains.

Fig. 7 shows the dynamic shear modulii obtained in the different apparatuses at the tenth cycle of various stress amplitudes. Here again, the results vary quite substantially from one device to another. Additional data obtained for two other water contents show similar trends.

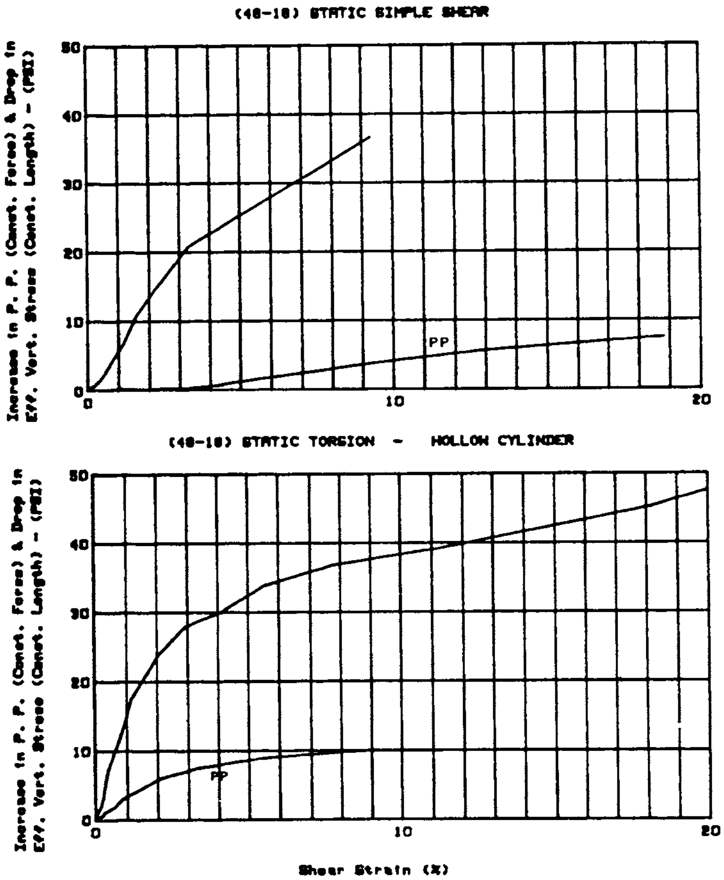

Fig. 5. Comparison of the Pore Water Pressure and the Drop in the Axial Effective Stress in the Simple Shear Device and in the Hollow Cylinder Device

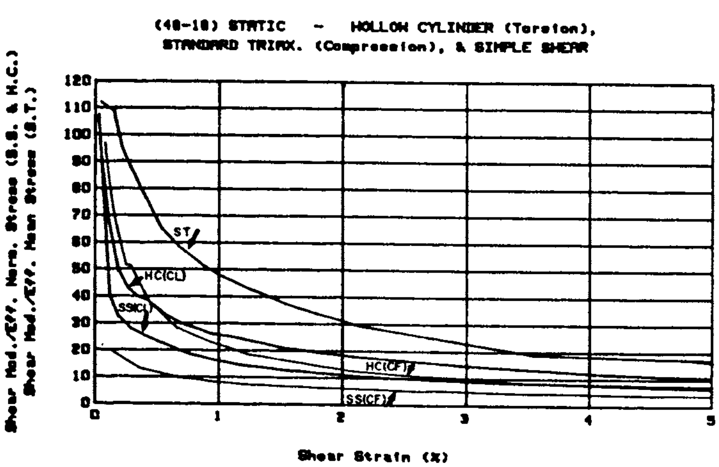

Fig. 6. Normalized Shear Moduli Obtained in the Standard Triaxial Test (ST), the Hollow Cylinder Test (HC) and the Simple Shear Test (SS) at Constant Volume or Length (CL) and Undrained or at Constant Force (CF)

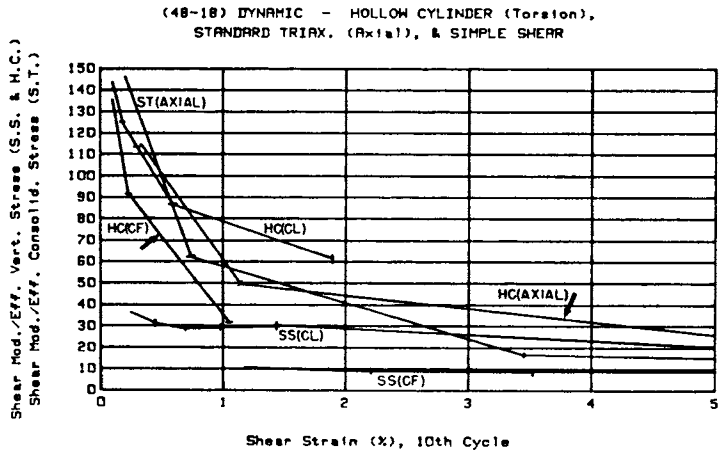

Fig. 7. Normalized Dynamic Shear Moduli Corresponding to the 10th Cycle and Various Levels of Strain in the Standard Triaxial Test, the Hollow Cylinder Test and the Simple Shear Test.

Conclusion

This discussion emphasizes the need for a better understanding of the meaning of the data obtained from the various devices. The need for security has led to very large safety factors and the scatter in field measurements can be used to justify the use of practially any apparatus. It all depends on the practitioner who has calibrated his mind to think in terms of a given test. Improvements of present design criteria, however, have to be based on research with devices that give a true picture of the mechanical properties of the material and not involve the device itself. If it is simple shear strain that a researcher needs, then the thin long hollow cylinder with the same inner and outer pressure and subjected to torsional stresses is the configuration to use. Any clay that can be cut with a piano wire will yield good hollow cylinders. In case of sands, researchers have used hollow cylinders for years; and while a little more time is required to prepare the samples, this preparation does not present any difficulty. It is obvious that there are many cases where one cannot obtain a hollow cylinder. In such cases the engineer turns to the most convenient device to obtain the so called "ball park value". The hope is that the "ball park value" approach will not slow down the tendency towards the use of more accurate devices in routine laboratory testing.

Acknowledgement

The research mentioned above is sponsored by the National Science Foundation to whom the writer wishes to express his deepest gratitude.

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Discussion by F.E. Richart, Jr.,
University of Michigan

Ten of the papers included in Sessions 1A and 1B showed relationships between the dynamic shear modulus and shearing strain and/or hysteretic damping and shearing strain. Seven of these papers included figures representing the abscissa as the numerical value of shearing strain, γ , whereas three of the papers used as abscissa the ratio γ/γ_r, where γ_r is the "reference shearing strain" (see Hardin and Drnevich), 1972, Richart, 1977, and Drnevich and Massarch, 1979, for example).

This disucssion is to reinforce the importance of relating the modulus reduction, G/G_o, and damping ratio, D, to the ratio γ/γ_r rather than to the simple numerical value of shearing strain. Figure 1 shows the hyperbolic shearing stress-strain curve described by Hardin and Drnevich (1972), and the definition of the "reference shearing strain", $\gamma_r = \tau_m / G_o$. For a cohesionless soil, the maximum shearing stress, τ , is a function of σ_o' , whereas G_o is a function of $(\sigma_o')^{1/2}$. Thus, for different confining pressures, different values of τ/τ_m or G/G_o should be anticipated for a given numerical value of shearing strain, .

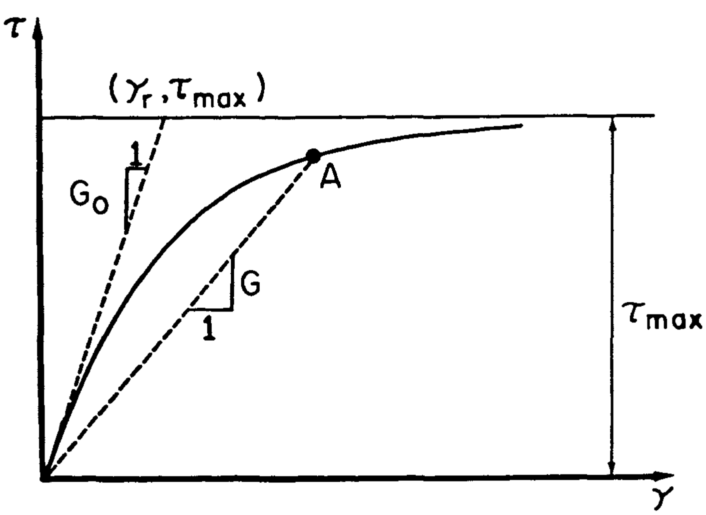

Fig. 1. Basic Parameters for Hyperbolic Shearing Stress-Shearing Strain Relationships for Soils: Reference Strain = $\gamma_r = \tau_{max}/G_o$.

To illustrate this point, Fig. 2 shows the modulus degradation, (G/G_o), vs. shearing strain amplitude, , for a silty sand under three different confining pressures. Three different curves are developed. This information can be represented by a modified Shibata-Soelarno (1975) curve having the form:

$$G/G_o = \cfrac{1}{1 + 315 \cfrac{(\gamma)\ 0.71}{(\sigma_o')^{0.5}}}$$

In order to present this information on a dimensionless basis, the values of τ_m were obtained as 0.29, 0.60, and 1.27 kgf/cm^2 for σ_o' = 0.43, 0.90, and 1.90 kgf/cm^2, respectively Then, the data can be represented as shown in Fig. 3, as a plot of G/G_o vs. γ/γ_r. Note that the test data now fall essentially on one curve This test curve can be represented by a Ramberg-Osgood curve described by the parameters shown in Fig. 3.

It is recommended that in the future presentations of test results describing G/G_o reductions, or damping increases with increasing shearing strain, be shown in terms of the dimensionless shearing strain ratio, γ/γ_r.

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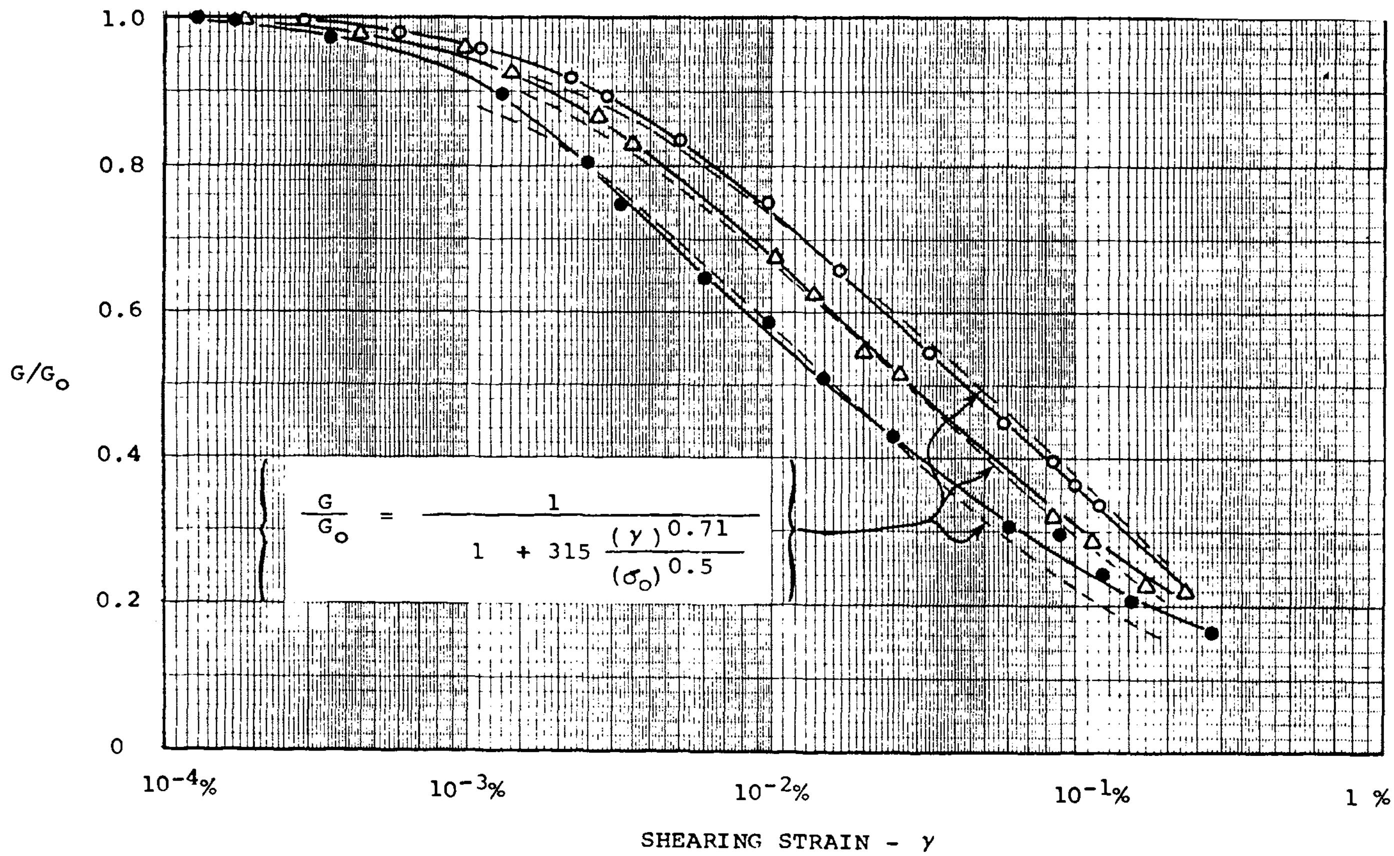

Fig. 2. Silty Sand, G/G_o vs. Shearing Strain, γ.
(○ = 1.90 kgf/cm^2 , △ = 0.90 kgf/cm^2 , ● = 0.43 kgf/cm^2)

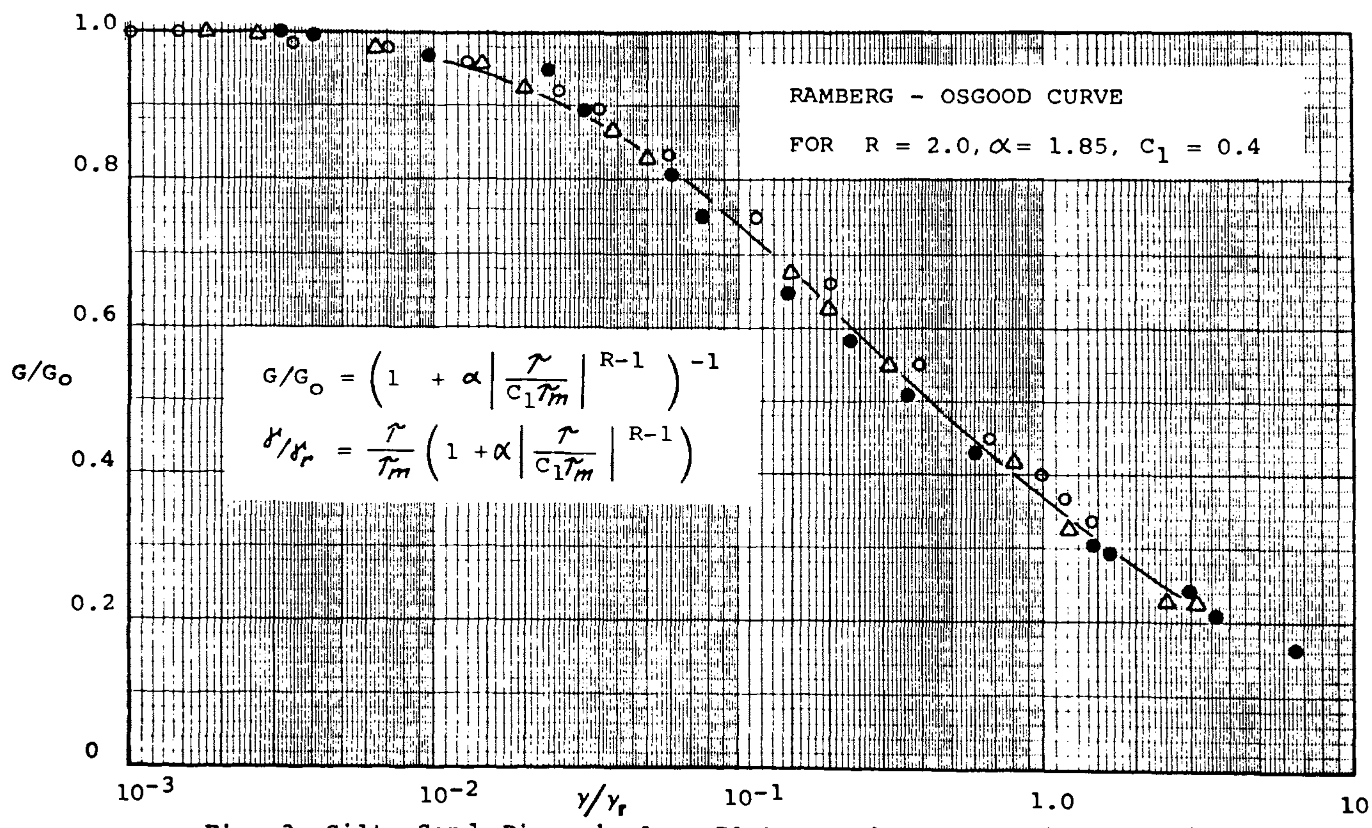

Fig. 3. Silty Sand, Dimensionless Plot of Soil Data - G/G_o vs. γ/γ_r
● = σ_o' = 0.43 kgf/cm^2 , △ = 0.90 kgf/cm^2 , ○ = 1.90 kgf/cm^2

Discussion by Tuncer B. Edil, Professor of Civil and Envrionmental Engineering, University of Wisconsin, Madison, Wisconsin

The response of soils to dynamic loading encompasses a wide range of behavior. Problems include stress-strain response and pore-pressure buildup and consequent loss of strength. Four papers to the conference, as sent to me, are discussed here.

Discussion of Manuscript "Predicting Pore Pressure and Applications to Complex Loading Problams" by D.A. Sangrey and V.P. Lascko

In this laboratory study of pore pressure response of soils subjected to undrained cyclic loading, the authors describe a procedure for predicting the development of excess pore pressure based on tests of a silty clay. The procedure involves the fitting of a hyperbolic equation to the normalized pore pressure versus number of loading cycles data. The authors depart from the previous reports of pore pressure buildup in the way they normalize the pore pressure. In the proposed normalization, excess pore pressure is divided by the maximum pore pressure potential defined as the pore pressure, at the critical level of repeated loading. The ordinate of Fig. 2 and how it differs from the ordinate of Fig. 3 is not clear. The normalization procedure is based on the premise that the shape of the pore pressure increase curve is nearly hyperbolic; however, the experimental data indicate that for sands and sometimes for clays, the curve will have a concave upward hook at the upper end. Examples of such curves are included in the proceedings of this conference (Vol. I, pages 62, 109 and Vol. II, pages 673, 674 for sands; Vol. I, pages 102 for clays). It is shown that the normalized relationship represents the data for the silty clay tested with an error of 35% in some cases. This is rather a wide margin.

An interesting application of the pore pressure prediction to sequencing (non-uniform load or strain history) is described. However, since no comparison with actual experimental results were provided, it is not clear how well this procedure works. One of the postulates of this method is that the response to a load cycle depends only on the effective stress at the time of the stress cycle application irrespective of the stress history. Verification of this postulate would be a significant contribution. Because of the irregular stress variation during earthquakes, means of predicting pore pressure buildup for such stress or strain histories are needed. It has been suggested by some investigators that the earthquake stress/time history can be represented by an equivalent uniform stress cycle. However, recently a number of methods are being proposed, such as the authors', to incorporate the irregular stress history directly to the formulation (for other examples in this conference refer to Vol. I, pages 107 and 231; Vol. II, pages 603 and 668).

The authors' desire to base the dynamic response analysis on effective stress methods is a fundamental one. Simplification of the procedure, which requires the prediction of pore pressures, is necessary for wide spread use of such methods.

Discussion of Manuscript "Pore Pressure in Silty Sand Under Cyclic Shear" by R.L. Wei, T.L. Guo and Y.M. Zuo

The authors report the results of their extensive cyclic loading tests on a silty sand observed to have liquefied during the Tangshan earthquake in 1976. The tests were performed on reconstituted specimens using a cyclic simple shear apparatus developed by them. It is indicated that undisturbed samples were taken; however, it is not clear how these samples were taken and why they were not used in the testing program but instead reconstituted specimens were used. Use of undisturbed specimens and comparisons with reconstituted specimens would provide significant insight especially in view of the fact that the material tested contained significant amount of fines. The fabric effects on the liquefaction behavior of such soils are expected to be more pronounced than the clean sands.

The tests reported were carried out with and without an induced initial static shear stress and, as has been established previously by a number of investigators, marked differences were observed in the behavior. There exists a certain amount of controversy regarding the effects of initial shear stresses. In the dynamic triaxial tests this issue is further complicated because of the superposition of two effects; namely isotropic versus anisotropic consolidation to simulate the in situ stress conditions and initial static shear stresses along potential failure surfaces. Simple shear tests reported in this paper provide additional support to certain contentions advanced by a number of investigators previously.

The test results are expressed in terms of normalized variables and empirical relationships developed by the authors. One of the interesting normalization terms is the "factor of dynamic effect, r_d" which combines the influence of the amplitude and number of cycles of the dynamic stress. It is reported that the behavior of this silty sand is similar to that of a medium sand in a qualitative sense. It would be very useful if some insight with respect to the quantitative effect of the presence of fines was provided.

Discussion of Manuscript "Dynamic Deformation Characteristics of a Soft Clay" by B.A. Andreasson

The author compares the results of field and laboratory determinations of the shear modulus of a soft, high-plasticity clay. Field tests included cross-hole, down-hole, and dynamic screw-plate loading tests and a high-amplitude resonant column device was used in the laboratory. Presented test results indicate near perfect correlation between the field and the laboratory (adjusted for long-term increases) shear wave velocities and between the screw-plate and the laboratory modulus reduction curves. In the comparisons, only selected results are displayed. For example, for field shear wave velocity, "the most reliable in situ measurement" was used. It would be interesting to see how different field tests compared. The type of field-laboratory correlation displayed for this soft clay may not be observed for other stiffer soils (see Vol. II, pages 591-596) because of the greater influence of unavoidable changes introduced by sampling and laboratory testing of such soils. One of the reasons why variations between different field tests as well as between the field and laboratory tests occur is the anisotropy encountered commonly in soils. Anisotropy may arise from different sources such as fabric anisotropy (particle orientation), stress-induced anisotropy, and directionality of the compression wave velocity in partially saturated soils. Laboratory investigation of the influence of fabric anisotropy on shear modulus and damping response of a clay carried by the discussor and his associates indicated a variation of 50% to 70% in dynamic shear modulus as a function of specimen orientation. In the same study it was shown that the variation of dynamic modulus with specimen orientation followed an elliptical relationship given as

$$G_{o_\beta} = \frac{A}{\sqrt{A^2 \sin^2\beta + \cos^2\beta}} \, G_{o_h}$$

where G_o = low-amplitude dynamic shear modulus, $A = G_{o_v}/G_{o_h}$ (v and h refer to vertical and horizontal specimens, respectively), and β = specimen orientation angle ($\beta = 0$ for vertical specimen, and $\beta = 90°$ for horizontal specimen). Shear modulus was determined using a resonant column apparatus applying torsional vibrations. Therefore, for the vertical specimens the torsional vibrations were applied in the bedding plane of the vertical specimen resulting in the lowest directional modulus. Such relationships can be utilized in determining the in situ anisotropy of soils and correlating the results of tests involving different wave propagation directions such as down-hole and cross-hole tests.

The author's modulus reduction curves plot above the one given by Seed and Idriss for clays. They would also plot above the curve for sands as given by Seed and Idriss. The underestimation of test results for clays by the Seed-Idriss curves (both for clays and sands) is observed by the discussor and the others (see Vol. II, page 598). The time of determination of the modulus after the application of the consolidation pressure is not indicated in Figures 8, 9, and 10. Since the modulus changes with time, meaningful comparisons are difficult without this information. Dynamic screw-plate loading test is an interesting one even though there is some difficulty in obtaining absolute modulus values due to the ambiguity in defining the shear strain. Improvement of field and laboratory techniques and development of consistent procedures in arriving at true in situ dynamic properties are important and attention to this problem is to be commended.

<u>Discussion of Manuscript "Soil-pile Interaction in Liquefiable Cohesionless Soils During Earthquake Loading" by H. Matlock, G.R. Martin, I.P. Lam, and C.F. Tsai*</u>

This paper differs from the previous three in its topic and emphasis. Nevertheless, it is somewhat related because it uses the effective stress methods and in situ dynamic properties in developing a procedure for soil-pile-structure interaction analyses under earthquake loading using basically two computer programs: DESRA and SPASM. DESRA utilizes the effective stress approach for free-field site response taking into account the generation, dissipation, and redistribution of pore pressures during earthquake loading. The initial backbone curve of soil p-y curve is degraded at each time step in accordance with the pore pressure increase (or the effective stress decrease) from the DESRA solution. This procedure definitely makes the p-y curves used in the soil-pile interaction analysis more realistic. Simplification and modelling of the structure can be improved. Interface element property evaluation and pile group interaction require further studies. Furthermore, in the application of the procedure to offshore structures, pore pressures induced in the seabed by traveling waves may be significant and should be taken into account. Recent advances in this area can be incorporated to the analysis in future. The procedure described by the authors is a definite improvement.

CONCLUSIONS

The areas of pore pressure generation under cyclic loads and the dynamic load-deformation response are still being researched actively. Accumulation of data from independent sources provides a generally consistent pattern for the laboratory behavior of soils. However, efficient and general empirical relationships are not apparent on a widely accepted basis yet. Application of the knowledge gained in the laboratory to the field behavior, on the other hand, appears to be lagging.

*Pertains to Session 4.

Discussion by R.C. Chaney, Associate Professor, Lehigh University, Bethlehem, PA on "Determination on "Determination of Dynamic Shear Modulus of Soils from Static Strength" by Y.S. Chae, W.C. Au, and Y.C. Chiang.

The authors have presented an interesting summary of previous work on the dynamic behavior of treated soils. In their paper the authors discuss both the effect of treatment level on the dynamic shear modulus and also present correlations between the dynamic shear modulus and static strength. The writer would like to discuss the correlation between the dynamic shear modulus, and static strength.

The equations presented in the paper correlating the maximum dynamic shear modulus (G_{max}) with static strength are of the following form.

$$G_{max} = m \bullet \bar{\sigma}_d + y \qquad (1)$$

where $\bar{\sigma}_d$ is the deviatoric stress and m,y are constants which are a function of soil type only as was pointed out by two of the authors in a previous paper (Au and Chae, 1980). Use of the above equation implies that both the treated and untreated soils generate similar stress strain curves. In addition the above relation also implies that the G/G_{max} versus log shearing strain (γ) curve for both treated and untreated soils are the same. Reviewing this second point then the following is true

$$G/G_{max} = \text{constant} = c \text{ at } \gamma = 1.4\% \; (\varepsilon_f = 1.0\%) \qquad (2)$$

$$\text{where } G = \frac{\tau}{\gamma} \quad , \quad \tau = \frac{\bar{\sigma}_d}{2} \qquad (3)$$

then

$$G_{max} = \left(\frac{1}{2c\gamma}\right) \bar{\sigma}_d = \frac{1}{2\gamma} d \; \bar{\sigma}_d = m \; \bar{\sigma}_d \qquad (4)$$

where $d = \frac{1}{c}$, $m = \frac{d}{2\gamma}$

For $\gamma = 0.014$, then m = 35.7 d. Comparison with the authors data as shown in Table 1 gives values of c corresponding to the range of values expected for G/G_{max} at $\gamma = 1.4\%$ as given by Hardin and Drnevich (1972) and Seed-Idriss (1971) for untreated soils. The significance of the y term in Eq. 1 must be connected in some manner with the way in which the individual soil particles are bonded together by the various additives and does not appear justified for untreated soils.

TABLE I. Summary of G/G_{max} Ratios at $\gamma = 1.4\%$ Based On Authors Results

Material	m	d	G/G_{max} at $\gamma = 1.4\%$
Sand	420	11.8	0.085
Silty-Clay	600	16.8	0.059
Clay	130	3.6	0.274

The last item the writer would like to present is additional data for the G_{max} vs $\bar{\sigma}_d$ curve for low deviatoric stresses. In Fig. 1 data is presented from undisturbed soft clay samples (γ_{dry} = 70 pcf) from the Gulf of Mexico. A review of this figure shows that the behavior is of the form described by Eq. 2. Therefore for untreated soils at low deviatoric stresses the relationship between G_{max} and $\bar{\sigma}_d$ is described by Eq. 4.

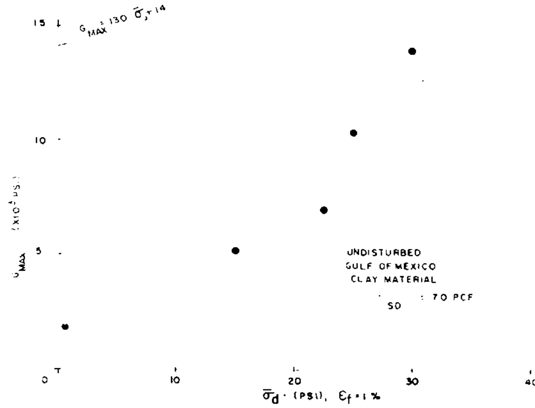

Fig. 1. Correlation Between G_{max} and Static Strength

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Discussion by R.C. Chaney, Associate Professor, Lehigh University, Bethleham, PA, on "Shear Modulus and Deformation of Soils Under Cyclic Loading, by Ting Hu.

The author has presented an interesting paper covering several topics concerned with the deformation of soils under cyclic loading. The writer would like to briefly discuss two of these topics topics which are (1) Effect of Shear Dilatancy on G of Sandy Soils, and (2) Deformation of Soils Under Cyclic Loading.

EFFECT OF SHEAR DILATANCY ON G OF SANDY SOILS

The author presents an interesting relation for correcting G for dilatancy effects in terms of two parameters m_1 and m_2. The writer finds this difficult to accept or reject because no data is presented to substantiate this relationship. Several observations can be made: (1) at low shearing strains (γ < 0.01%) most soils in the writers experience behave elastically with no volume change, and (2) Eq. 5 as it is now formulated does not appear to be strain-dependent although it is well known that G is a function of the shearing strain level, γ . Therefore without backup data the writer is somewhat skeptical of Eq. 5 at the present time.

DEFORMATION OF SOILS UNDER CYCLIC LOADING

The author presents an interesting approach to the problem of the response of dry and partially saturated soils to cyclic loads. The writer would like to discuss the use of the cyclic triaxial device to determine the required material parameters. The theoretical total stress conditions for conventional cyclic axial triaxial tests on isotropically consolidated saturated specimens have been described by Seed and Lee (1966). This methodology has been extended by Chaney (1978a, 1978b) to dry and partially saturated soils by cycling both axial and lateral stresses 180^{o} out of phase with each other. This technique has been verified by comparing the results of cyclic tests on saturated specimens with data from conventional cyclic triaxial tests. Close agreement was shown to occur. Typical results for dry sand using this axial-lateral cyclic triaxial apparatus is presented in Figs. 1 and 2. If only an axial cyclic load is applied to the specimen the required alternating shear stress is not developed on a plane through the specimen.

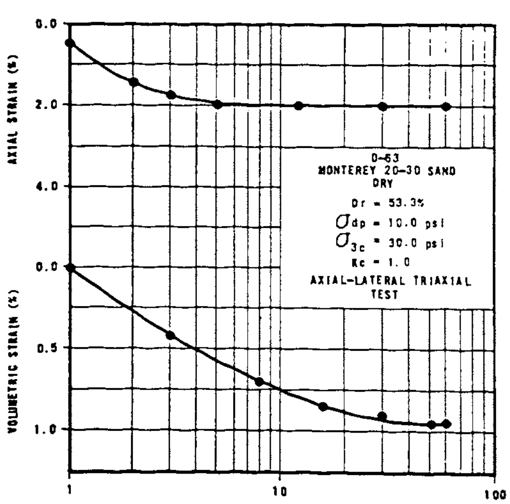

Fig. 1. Axial and Volumetric Strain Versus Number of Load Cycles, Monterey 20-30 Sand, Dr = 53.5%, σ_{3c} = 30.0 psi, K_c = 1.0.

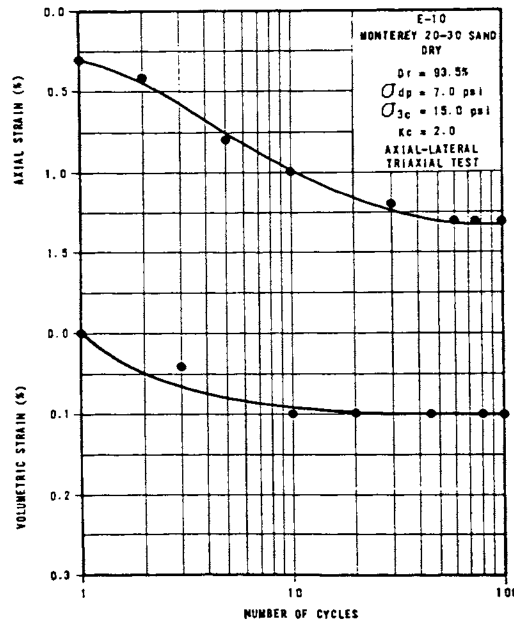

Fig. 2. Axial and Volumetric Strain Versus Number of Load Cycles, Monterey 20-30 Sand, Dry, Dr = 93.5%, σ_{3c} = 15.0 psi, K_c = 2.0.

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Discussion by Knut H. Andersen, Norwegian Geotechnical Institute, Oslo, Norway on "Pore Pressure Generation during Variable Cyclic Loading" by Dyvik, Zimmie and Schimelfenyg.

In Session 1B, Dyvik, Zimmie and Schimelfenyg present an interesting paper on the prediction of soil behaviour under undrained variable cyclic loading. Their prediction is based on the assumption that the soil behaviour at any stage during cyclic loading is governed by the pore pressures generated during the previous cyclic loading history. Dyvik et al. present test results on normally consolidated clays which support that assumption. This discussion presents some supplementary results and comments.

Smits, Andersen and Gudehus (1978) previously used the same assumption to evaluate the undrained behaviour of medium dense sand during variable cyclic wave loading. They also found reasonably good agreement between calculated and measured behaviour of simple shear tests with variable cyclic loading. Fig. 1a shows the principle for determining the pore pressure development. Further, Figs. 1b and 1c illustrate how the procedure may be extended to predict shear strains during variable cyclic loading. Both pore pressure and strain-contours are determined from stress-controlled tests with constant cyclic shear stress amplitude. Fig. 1b shows contours of cyclic shear strain. Fig. 1c shows contours for permanent shear strains. Permanent shear strains will develop in simple shear tests with unsymmetrical cyclic load and in triaxial tests. At any stage the equivalent number of cycles giving the same pore pressure generation as the previous cyclic load history, can be determined (point A in Fig. 1a). With this equivalent number of cycles, the corresponding cyclic and permanent shear strains can be determined from Figs. 1b and 1c, respectively.

Comparisons between calculated and measured behaviour for two of the tests with variable cyclic loading are presented in Figs. 2 and 3, and a reasonably good agreement is found. By extending the procedure to give strains in addition to the pore pressure development, the variation of soil modulus ($G = \tau/\gamma$) under variable cyclic loading may also be easily obtained and may provide the input to finite element analyses of stresses and displacements in the foundation.

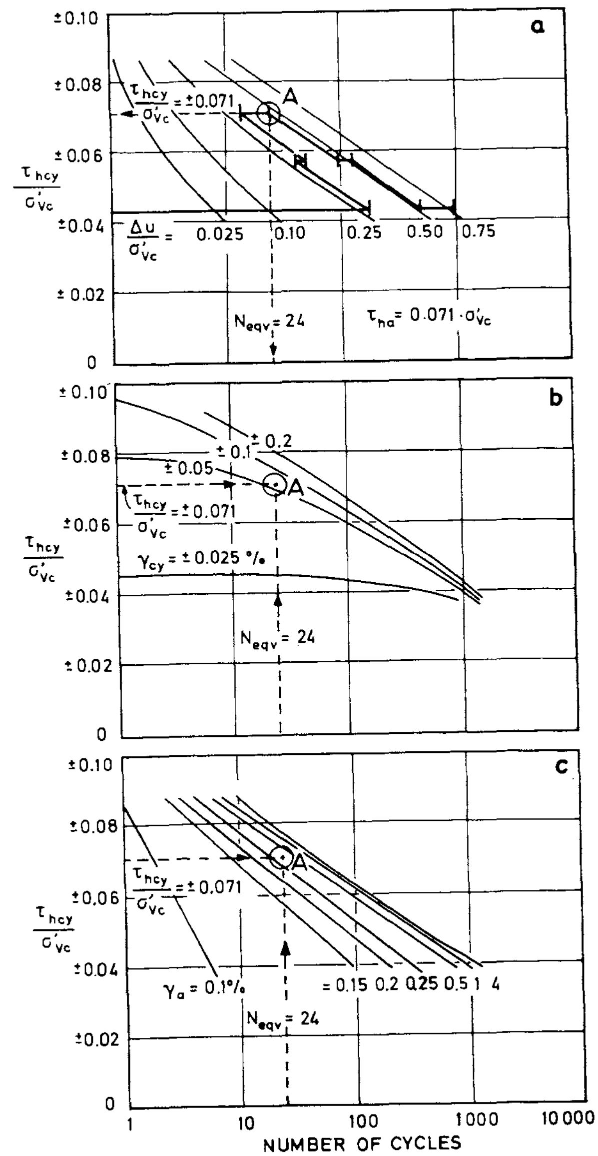


Fig. 1. Prediction of excess pore pressure and average and cyclic shear strains in test with varying cyclic shear stresses.

a) Development of excess pore pressure and determination of equivalent number of cycles.

b) Determination of cyclic shear strain, γ_{cy}

c) Determination of average shear strain, γ_a

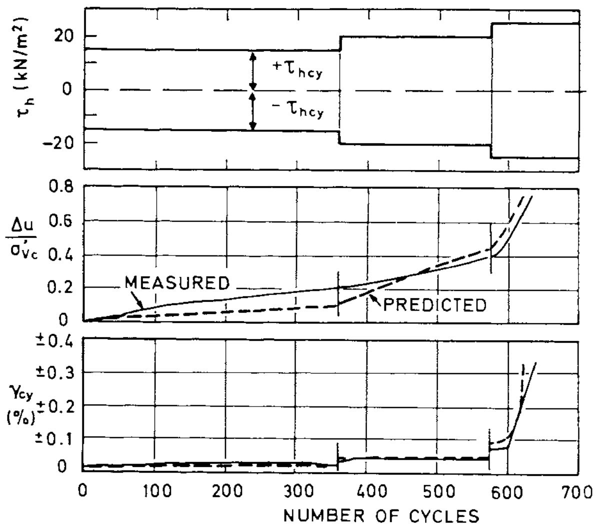
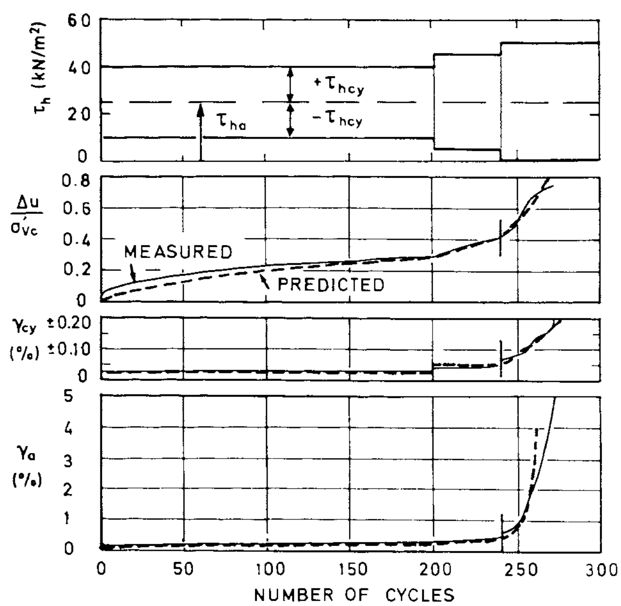


Fig. 2. Comparisons between calculated and measured excess pore pressure and strains for a simple shear test with varying cyclic shear stress amplitudes. Symmetrical shear stress.



As an additional control of the method, it would be valuable to investigate whether it is able to predict the behaviour of strain-controlled tests. This was used as a control of the strain approach which works with cyclic strain as the key parameter instead of pore pressure (Andersen, 1976; Andersen et al. 1978).

Dyvik et al. (1981) argue that the pore pressure approach is simplerthan the strain approach. This may be true if the purpose is to determine the pore pressure development and the number of cycles to failure. Further, it may be an advantage to work with pore pressures if the situation to be analysed is not undrained and drainage has to be taken into account. However, as emphasized by professor Silver in his state-of-the-art lecture in Session 1A, pore pressure measurements in cyclic laboratory tests are extremely difficult and demanding. If the purpose of the testing is to determine shear moduli and strength parameters to calculate displacements and stability of a foundation under undrained conditions, it may therefore be more reliable to use the strain-approach directly instead of introducing the unneccessary uncertainty inherent in the pore pressure measurements.

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Discussion by J.M.O. Hughes, Situ Technology Inc., Vancouver, B.C. on "In Situ Measurement of Dynamic Modulus and Damping of Pleistocene Soils" by H. Mori and H. Tsuchiva.

H. Mori and H. Tsuchiva in the paper on "In Situ Measurement on Dynamic Modulus and Damping of Pleistocene Soils" present the results of dynamic pressuremeter tests for use in determining the soil modulus at large strains. Similar studies have been going on in Canada in conjunction with the University of British Columbia, using a Self-Boring Pressuremeter similar to the instrument described by Wroth-Hughes (1973).

Because of the complex drainage situations and pore pressure developments under quick cyclic loads all these studies have been conducted under slow cyclic conditions so as near as possible drained conditions exist, i.e. - the boundary conditions are more clearly defined.

The results of two typical tests conducted in both loose and dense sands are shown in Figures 1 and 2 respectively. The modulus, as defined by the slope of the axis of the hysteresis loop, is clearly defined.

What is probably more significant is the degradation from cycle to cycle. The degradation of the loose sand is far greater than for the dense sand. The results are summarized in Figure 3.

The anticipated pore pressure which can be developed will be a function of the cyclic degradation and the permeability of the soil.

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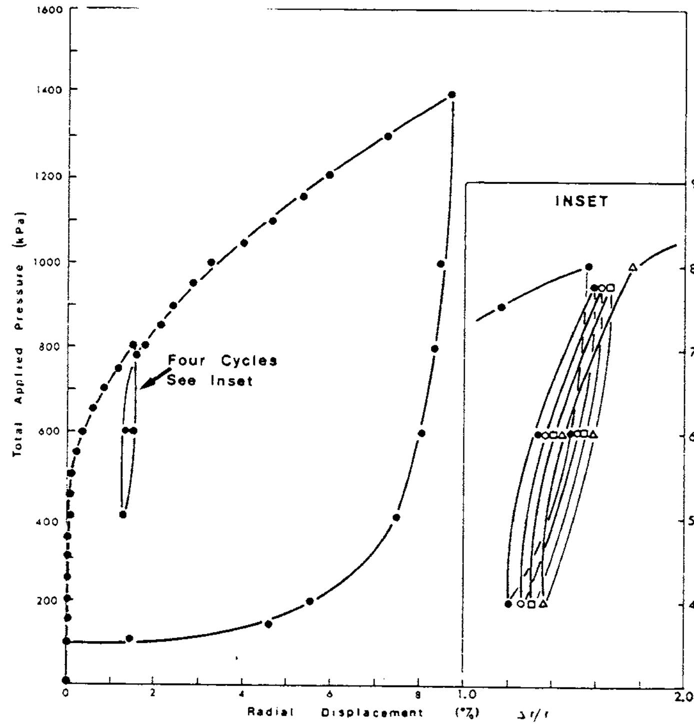
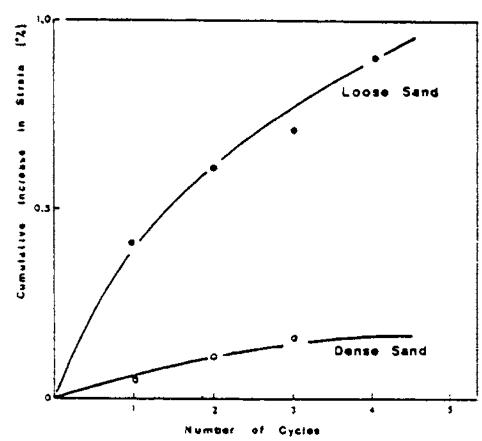
Wroth, C.P. and Hughes, J.M.O., (1973). An instrument for the In Situ Measurement of the Properties of Soft Clays. Proc. 8 International Conference on Soil Mechanics and Foundation Engineering, Moscow, Vol. 1, 487-494.



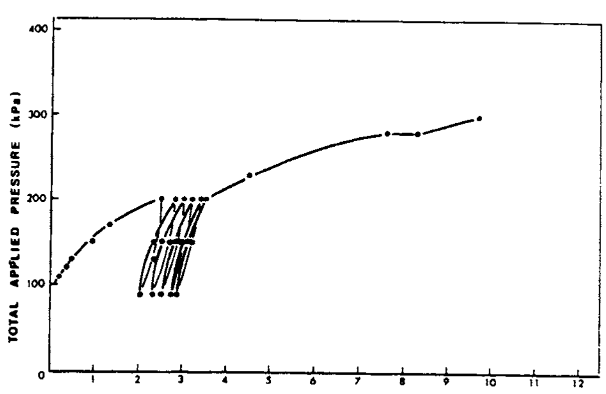
Cyclic test in dense sand

Fig. 2



Cumulative strain under several cycles of loading

Fig. 3



Cyclic test in loose sand

Fig. 1

Discussion by F. Oka, Associate Professor, Department of Civil Engineering, Gifu University, 3-1 Naka-Monzencho, Kakamigahara, Gifu, Japan, on "Cyclic Responses of Cohesive Soils Using a Bounding Surface Plasticity Model," by Y.F. Dafalias and L.R. Herrman

In relation to the liquefaction phenomena, is it possible to modify your Bounding Surface Model to predict the liquefaction of sand deposits Especially, how you describe the behavior of sand after the stress state attains to the critical stress ratio defined by phase transformation angle proposed by Ishihara et al.?

AUTHORS' REPLIES

Closure by Y.S. Chae, W.C. Au and Y.C. Chiang

The authors wish to thank Professor Chaney for the discussion. The simple relationship given to correlate the dynamic shear modulus with static strength, as expressed by Eq. 4 in the paper, was based on linear regression analysis of 162 test data points obtained over a very wide range of deviatoric stresses of up to 400 psi. If one should examine the actual plots (Figs. 7, 8 and 9) of the test results, however, this linear relationship exists, as pointed out in the paper, with an exception of those soils having very low strength. There is no question that the lines must go through the origin. However, it is not quite certain, experimentally at least because of the limited number of test data points, whether the initial slope m is constant or stress-dependent. Furthermore, as the deviator increases, the slope assumes a new, lower value. These observations made the authors to note in the paper that the linear relationship obtained was "for high-strength soils, and a further study is needed to evaluate the correlation, if any, for low-strength soils." Although the discusser shows the relationship to be linear (Fig. 1 in the discussion), the straight line was drawn through only five test data points obtained for the deviatoric stress of up to 30 psi. Whether the slope remains constant beyond the 30 psi deviatoric stress is not certain. It is quite possible that the relationship between dynamic shear modulus and static strength is linear, but with two different slopes. This reinforces the authors original contention that further research is needed to ascertain the relationship between the dynamic shear modulus and static strength of soils for low-strength soils.

Closure by R. Dyvik, T.F. Zimmie, and P. Schiemlfenyρ

The authors would like to thank Andersen for his excellent discussion of our paper. It is interesting to see that the pore pressure approach to modelling cyclic tests with varying cyclic stress levels also works reasonably well for sands. Andersen also reemphasizes the point made several times during the conference that pore pressure measurements in cyclic laboratory tests may not be very accurate. This is quite true and could present a problem when extrapolating laboratory results to soil behavior in situ. However, this may be lesser of a problem when predicting other laboratory tests using the same measurement procedures and equipment .

There is however, one difference between strain and pore pressure approaches to modelling cyclic behavior, which can best be shown by the following test results obtained by the authors. Two tests with varying cyclic shear stress levels were performed on the Pacific Illite. Test number 11 was cycled 500 times at a stress level (τ_c/S_u) of 37 percent and then changed to 51 percent. The "immediate change in cyclic shear strain," as predicted by the strain approach, was exactly the same as was measured in the change (the shear strain increased 0.08 percent). In test number 8, the cyclic stress level was changed to 37 percent after 563 cycles at 51 percent. The strain approach predicts a decrease in shear strain of 0.08 percent for this change of stress level (as before), but the actual shear strain went from 1.92 percent to 1.47 percent, which is a decrease of 0.45 percent. The reason for this discrepancy between predicted and measured change in strain is shown in Figure 1. This is the normalized shear modulus versus number of cycles for both tests, with each curve ending at the point of stress change. As can be seen in this figure, the modulus curves for both test are fairly flat initially, but then the curves for test 8 drops rapidly as the strains get larger and failure becomes imminent. The shear modulus at the end of these curves is very different and one would expect the change in shear strain to be also quite different for a similar change in stress level.

Therefore, the strain approach works very well until the modulus of the soil starts deviating significantly from the modulus in the first cycle. This means that there are situations where predictions based on parameters from the first cycle, such as the shear strain, may not be representative of the current behavior. Using pore pressure, which is a current parameter, does not present such a problem. In summary, there are many situations where both models seem equally valid, while there are others where one model may work better than the other, and should therefore be selected. Which model is used will depend on the problem at hand.

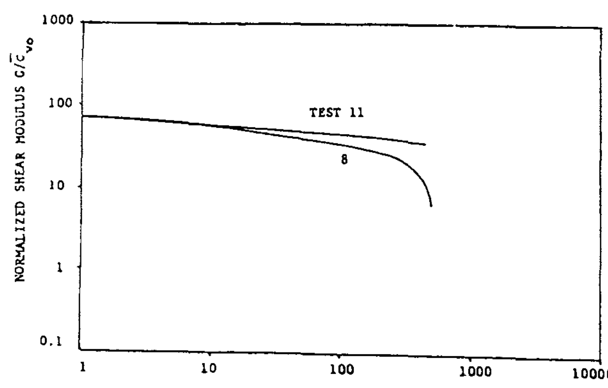


FIGURE 1 NORMALIZED SHEAR MODULUS VERSUS NUMBER OF CYCLES - PACIFIC ILLITE

AUTHOR'S REPLY

Closure by H. Mori.

The aim of dynamic pressuremeter tests is to obtain the equivalent shear modulus and the equivalent damping ratio for the non-linear stress-strain behavior of soils under undrained conditions. Therefore, the quick cyclic loading is applied assuming undrained deformation of soil around a probe. From the authors' experience that the amplitude of volume variation does not change within 15 cycles, the influence of drainage on the derived parameters is considered insignificant.

The degradation from cycle to cycle as pointed out by Dr. Hughes is recognized also in dynamic pressuremeter tests as indicated in Fig. 1.

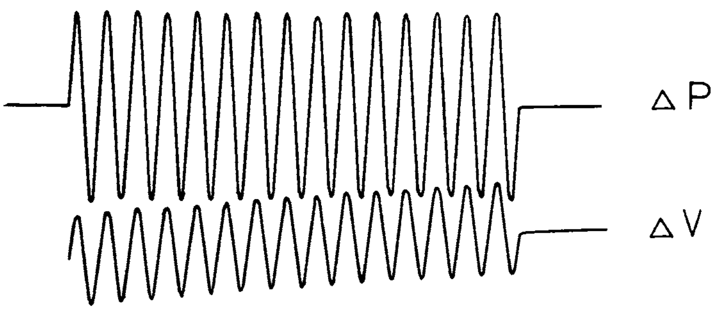


Fig. 1 Oscillographic records of dynamic pressuremeter test for dense sand

The amount of degradation depends not only on properties of soil but on (1) initial static state of stress and (2) cyclic stress (pressure) amplitude. The cyclic pressure is applied at the initial static pressure of $p_f - p_o/2$ (p_f : yield pressure), while the cyclic pressure illustrated in Fig. 2 of the discussion appears to start from a pressure exceeding p_f. The degradation tends to increase as the sum of static and cyclic pressure approaches to p_f.

In Fig. 2 is shown an example of degradation derived from tests in the Pleistocene sand (N-value of SPT about 50). The degradation at the 3rd step of loading is negligible, and it is in the order of 0.01% at the 9th step.

The degradation becomes a source of error when the equivalent shear modulus is to be applied to the total stress analysis. While, the shear modulus and volumetric strain to be applied to the effective stress analysis may be obtained from drained tests with the measurement of degradation as suggested by Dr. Hughes or undrained cyclic loading tests with pore pressure measurement. However, number of problems are left for research in the future in order to apply the effective stress analysis into practice.

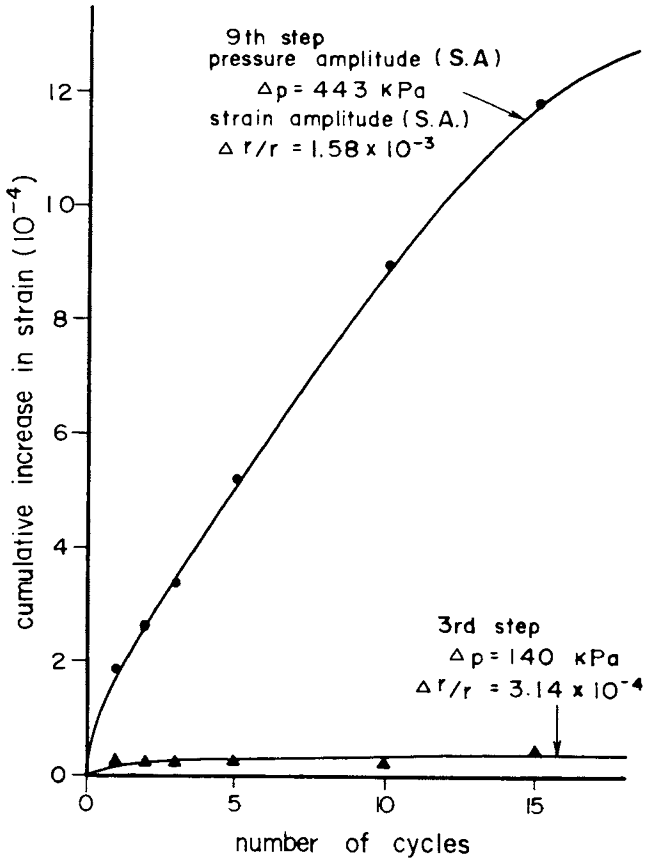


Fig. 2 Cumulative strain under cyclic loading

AUTHOR'S REPLIES

Closure by Y. F. Dafalias.

The concept of the Bounding Surface is a general one and does not apply to cohesive soils only. As a matter of fact, it was originated in an effort to describe the monotonic and cyclic response of metals. In the various references given at the end of the paper under discussion, it has been shown that in principle any rigorous classical yield surface plasticity formulation can be easily transformed into a corresponding bounding surface formulation, and this conclusion applies to sands as well. To this end, one should not try to modify a model which is appropriate for clays but rather take any particular plasticity model which applies to sand (e.g. Ishihara's, Lade's, Nova's, etc.) and generalize its formulation within the general framework of the bounding surface concept. At this point the authors are in the process of developing such generalizations for different classical yield surface plasticity models for sands, but elaboration on further details pertinent to each particular model (e.g. the phase transformation angle) are deferred for the near future.

Closure by Bo Andreasson.

A couple of questions/comments to my above paper have been raised by Professor Edil in Session 1A and by the moderator of this session, Professor Woods. The questions are related to:

(1) effects of anisotrophy on in-situ shear wave velocity.

(2) time of consolidation in the presented high-amplitude resonant column tests.

(3) method of interpretation of the dynamic loading screw-plate tests.

Anisotrophy: The tested soil is a soft, high-plastic marine clay and the effects of anisotrophy on the shear wave velocity are small. Shear wave velocities evaluated from cross-hole and down-hole tests do not show any significant difference, however, the down-hole tests show the largest scatter. The in-situ shear wave velocity compared with the laboratory shear wave velocity in Fig. 5, page 67, was deduced from cross-hole tests with three boreholes, a steady-state vibration source and the test procedure presented in the paper, page 66.

Consolidation Time: The consolidation time in the resonant column tests before execution of high amplitude tests, such as presented in Figs. 8-10, was about one week throughout the tests. (The in-situ dynamic loading screw-plate tests were carried out one day after installation of the screw-plate).

Interpretation of Screw-Plate Test Results: The lumped parameter approach was used to interpret the dynamic loading screw-plate test results. This is a simplification, but still believed to be a viable approach. More refined methods are certainly required to get a full understanding of the test method. This problem will be further investigated in Sweden in the near future.

Closure by Vasudevan Rajaram.

The size effect is very important in rock mechanics since rock in-situ has several discontinuities which significantly reduce rock strength. However, the basic understanding of rock behavior can be initially gained by testing laboratory specimens of relatively homogeneous rock. The laboratory results are then reduced by a factor of safety which is dependent on the extent of discontinuities (fractures, joints, etc.) that may be present in-situ. After access is gained through boreholes, or preferably a test excavation, in-situ tests can be conducted to determine rock mass behavior. In-situ testing of rock is expensive and in the initial stages of a project, the design engineer has to use laboratory test results and temper these with his judgement of in-situ conditions.

AUTHOR's REPLIES

Closure by Pan Fu-lan.

I would like to make some explanations and answer some questions on the "Analysis of Variation of Poisson's Ratio with Depth of Soil".

1. Relationship between shear modulus G, elastic modulus E, shear velocity V and coefficient of subgrade reaction C and depth of soil was discussed in another paper published in "Acta Mechanica Solid Sinica", No. 1, 1981.

2. In the past, the propagating velocity of elastic wave, elastic modulus, shear modulus, Poisson's ratio etc. of homogeneous soil were all supposed constants, without considering the influence that laying depths have on these coefficients. A great many field test informations show that the above coefficients increase with the depth. Why so? The reason for this completely different result is due to different testing methods. Tests performed in laboratories change the boundary conditions of the soil. On the other hand field tests keep the natural form of the soil and boundary conditions intact. Especially as the influence of compressive stress caused by the dead weight of soil body is the fundamental reason for the change in coefficients. This is just the point where soil mechanics differs from elastic mechanics.

3. The wave velocity, elastic modulus, etc. of hard soil was regarded to be greater than that of soft soil. This is not a thorough way of putting it. It is necessary to point out the depth of soil. Only then will it have practical meaning. A great many tests show: Wave velocity of the same soil, due to different laying depths will differ greatly. The wave velocity of soft soil at deep strata might be far greater than that of hard soil at shallow strata. Thus only different kinds of soil at the same depth can be compared.

4. Ratio of lateral compressive stress to vertical compressive stress acted upon an element increases with the increase of depth. But the lateral compressive stress increases faster than vertical compressive stress. Therefore, at a considerable depth of soil, the lateral compressive stress tends to be equal to the vertical compressive stress step by step. This shows that at a considerable depth the soil body is incompressible. Taking clay loam for example: When using shallow strata soil sample, the ratio of lateral compressive stress and vertical compressive stress as 0.4 to 0.6, the result is normal. But when using deep strata soil sample, with the same ratio of lateral compressive stress and vertical compressive stress, the sample is destroyed. Only when the ratio of both approach to 1, you get a normal result.

5. Professor D.D. Barkan assumes that Poisson's ratio decreases with the increase of depth. This point is quite opposite to the tests and theoretical analysis I have made. As a matter of fact, on condition that the dead weight of soil body influencing shear modulus is under consideration, the vertical strain and horizontal strain decreases with the increase of depth under the action of an unit compressive stress.

But the vertical strain decreases faster than the horizontal strain, thus, Poisson's ratio of homogeneous soil increases with depth of soil. At a considerable depth, its value approaches to 0.5. This also proves that deep strata body is incompressible. Of course, tests made on body taken from deep strata and under natural conditions will give you completely different results. Thank you.

Closure by D.H. Timmerman and S. Leelanitkul.

The data presented in Fig. 4 in the original paper (Vol. 1, p. 137) supposedly shows progressive strain-vs-shearing strain amplitude from cyclic triaxial testing. The data shown is actually from cyclic simple shear tests conducted by Seed and Silver (1971). The correct data from the cyclic triaxial testing is shown below as Fig. 4a.

The similarity between the original Fig. 4 and the correct Fig. 4a further supports the procedure proposed by the authors for determining progressive strain of sand under cyclic loading as a procedure which is independent of the loading conditions being considered.

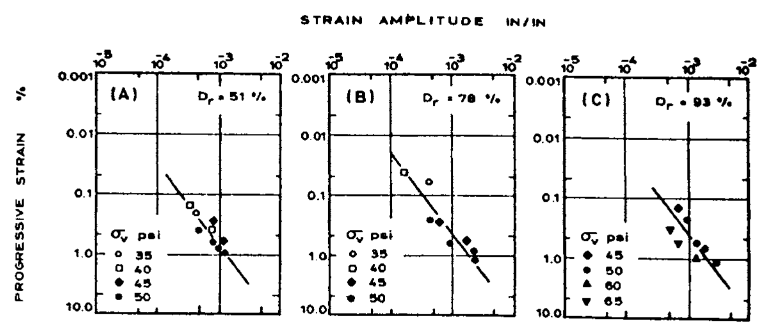


Fig. 4a. Progressive Strain-vs-Shearing Strain Amplitude

REFERENCE

Silver, M.J. and H.B. Seed (1971), "Volume Changes in Sands During Cyclic Loading", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM9.